Performance Evaluation of Pile Foundation Using CPT Data

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SYNOPSIS: The Southern California Edison Company (SCE) has utilized cone penetrometer test (CPT) data for design of concrete drilled shafts and driven piles at various facilities over the plast seven years with substantial cost savings in field exploration and foundation design. This paper incorporates a performance evaluation of drilled shafts and driven piles to predict the uplift load versus deflection curves based on the embedded length to diameter ratio of each foundation. A revised design methodology is presented to correlate the side friction values from CPT data with field uplift load test results in granular and cohesive soils.

INTRODUCTION

The Southern California Edison Company (SCE) has utilized cone penetrometer test (CPT) and exploratory boring results for design of concrete drilled shafts and driven piles along transmission line routes and other facilities within its service territory over the past seven years. Substantial cost savings have been achieved in field exploration and foundation design.

A performance evaluation was made incorporating SCE uplift load tests performed over the past 50 years. Normalized curves are presented which utilize the peak uplift resistance or ultimate uplift capacity at a vertical displacement of one inch (2.5 cm). Thus, the uplift capacity of drilled shafts and driven piles may be predicted with associated deflections based on the shape of the foundation.

A revised design methodology is presented in this paper for use with CPT side friction values. Correlation charts are given for drilled shafts and driven piles to obtain the total side friction resistance of foundations in granular and cohesive soils.

LOCATION OF FIELD LOAD TESTS

A total of 30 uplift load tests were performed at 21 sites within the SCE service territory. The location of three transmission lines, Magunden-Pastorja, Mira Loma-Serrano and Devers-Palo Verde are shown in Fig. 1, along with the Westminster facilities. Also, nine uplift load tests were conducted along the Intermountain Power Project (IPP) transmission line at four sites in Utah, Nevada and California by the Los Angeles Department of Water and Power (LADWP).

SOIL CONDITIONS

The soil conditions at the four test sites along the Magunden-Pastoria transmission line, southeast of Bakersfield, consisted of loose to medium dense silty sands and silts. Along the Mira Loma-Serrano transmission line, these same conditions were encountered at Sites 4XX and 14AX. Sites 17 and 29X were comprised of surficial silty sands overlying intermixed silty sand and clay layers to a depth of 30 feet (9.1m). Site 27 consisted of medium stiff clay

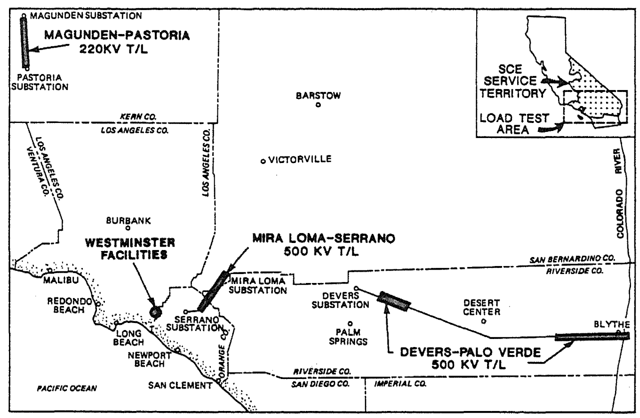


FIGURE 1. LOCATION OF SCE UPLIFT LOAD TESTS

to a depth of 15 feet (4.6m). At site 159, a stiff clay stratum was noted from 3 to 6 feet (0.9 to 1.8m) overlying dense sands with gravel to a depth of 10 feet (3.0m).

During construction of the Devers-Palo Verde transmission line, loose dune sands were encountered at Sites 4018 and 4107, east of Palm Springs. Extreme caving was noted at Site 4018, so a cement slurry was placed in the excavation and the test pier was constructed by drilling through the slurry to the design depth. Between Sites 4722 to 4747, west of Blythe near the Colorado River, the subsurface conditions consisted of loose silty sands overlying intermixed sand and clay strata to depths up to 15 feet (4.6m). Medium dense to dense sands were encountered at depths from 15 to 40 feet (4.6 to 12.2m). Groundwater was located from 10 to 15 feet (3.0 to 4.6m) below the ground surface in this area.

At the Westminster facilities, medium dense silty sands were encountered in the upper five feet (1.2m) overlying intermixed silty sand, silt and clay layers to a depth of 25 feet (7.6m). From 25 to 60 feet (7.6

to 18.3m), dense sand strata were encountered with thin layers of soft clay at various depths. These dense sand layers required predrilled holes so that the precast concrete piles could be driven to the design depths.

Along the Intermountain Power Project transmission line, the Delta and Alamo sites in Utah consisted of stiff to very stiff overconsolidated clay. The Caliente site in Nevada and Baker site in California were characterized by cemented silty sands overlying sands and gravels to a depth of 20 feet (6.1m). Detailed soil conditions for the IPP sites along with pressuremeter data are presented by Briand, et al. (1984).

CONE PENETROMETER TEST RESULTS

The CPT soundings were performed by Earth Technology Corporation and Pioneer Consultants using a standard electric cone pushed at a rate of 0.8 in/sec (2 cm/sec) using a 20 ton (178 kN) reaction truck. Both side friction and point resistance profiles were recorded continuously and used in computing the friction ratios.

UPLIFT LOAD TEST PROCEDURES

Uplift load tests were performed on 30 drilled shafts and driven piles by SCE personnel using a portable steel tripped test frame. This frame is 10 feet (3.0m) high and has three legs spaced 18 feet (5.5m) apart at 120 degree angles from each other. A double acting hollow plunger hydraulic jack with 150 ton (1335 kN) capacity and 8 inch (20.3 cm) stroke was used to apply the tensile loads. The jack has a 3 1/8 inch (7.9 cm) diameter hole through its center and rests on a 1 inch (2.5 cm) thick steel plate at the top of the tripod. A 1 3/8 inch (3.5 cm) diameter, high strength Dywidag bar extends through the jack and was attached to the top of the foundation.

Load tests were conducted by applying a tensile load to the Dywidag bar in increments of approximately 25 percent of the design load. The load was rebounded to zero from 25, 50 and 75 percent of the design load, then the load was re-applied until failure was reached prior to a final rebound, if possible.

Deflections at the top of the test foundations were measured by the use of generally three dial gauges, with an accuracy to 0.0001 inch (0.00025 cm), located at 120 degree spacing around the circumference of the top of the pile. The dial gauges were mounted to a rigid frame which was supported outside the perimeter of the load test frame.

BASIC CONSIDERATIONS

In principle, the uplift capacity of drilled piers in granular soils is shown in Fig. 2a and may be computed from the following vertical equilibrium equation:

$$Q_u = W + Q_s + Q_t \qquad (1)$$

with Q_u = ultimate uplift capacity, W = foundation weight, Q_s = side resistance and Q_t = tip resistance. The side resistance varies depending on the shearing surface and shearing resistance of the granular materials. The tip resistance can be developed from tension and suction stresses at the bottom of the foundation. During drained loading, suction is not present and tip tension is normally very low for cast-in-place concrete drilled shafts as described by Kulhawy (1985). Since the tensile

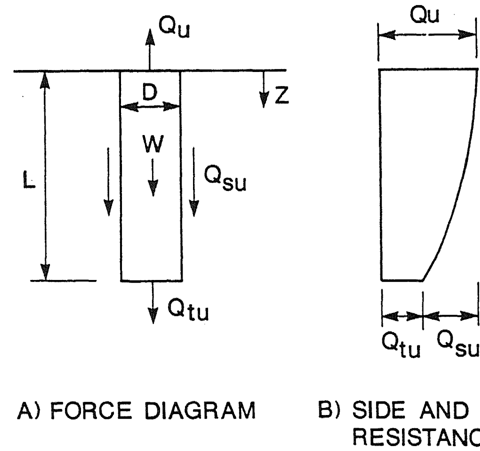


FIGURE 2. DRILLED PIER IN UPLIFT

strength of granular soils is usually low, the tip resistance for the drilled shafts and driven piles was assumed to be zero.
The side resistance, Q_s, is shown in Fig. 2b and may be expressed as:

$$Q_s = Q_u - W - Q_t = \int_0^L (A_s)(f_s)\, dz \qquad (2)$$

where A_s = Surface area of soil-shaft interface, f_s = Average skin friction along soil-shaft interface and L = Embedded length of foundation. The side resistance varies in a parabolic manner along the shaft to a minimum value at the tip of the shaft based on load test results from Reese, et al. (1976), Vesic (1970) and others.

For cast-in-place concrete drilled piers, the soil-shaft interface occurs adjacent to the perimeter of the shaft. Along transmission line routes, belled piers are typically used due to increased uplift resistance as compared to drilled piers of the same length. The generalized failure surfaces for belled piers are shown in Fig. 3a depending upon the embedded length to shaft diameter (L/D) ratio. An equivalent force diagram using a cylindrical shear failure surface is shown in Fig. 3b for belled piers in normally consolidated soils where the uplift "breakout cone" is not likely to develop based on recent studies. The mean diameter for belled piers may be obtained from the following relationship:

$$D_{mean} = D_{shaft} + 1/3\,(D_{bell} - D_{shaft}) \qquad (3)$$

where D_{shaft} = Average shaft diameter and D_{bell} = Diameter at base of pier.

FOUNDATION PERFORMANCE EVALUATION

The performance of drilled shafts and driven piles have been described by many authors in various soil conditions. For this study, field uplift tests by SCE were evaluated based on the foundation geometry (L/D ratios) where the peak capacity was reached. When the peak capacity occurred at larger displacements, the ultimate capacity was selected at a vertical deflection of one inch (2.5cm). This deflection criteria has been used by SCE in design of foundations for transmission line towers and substation steel structures after tower failures occurred in high winds.

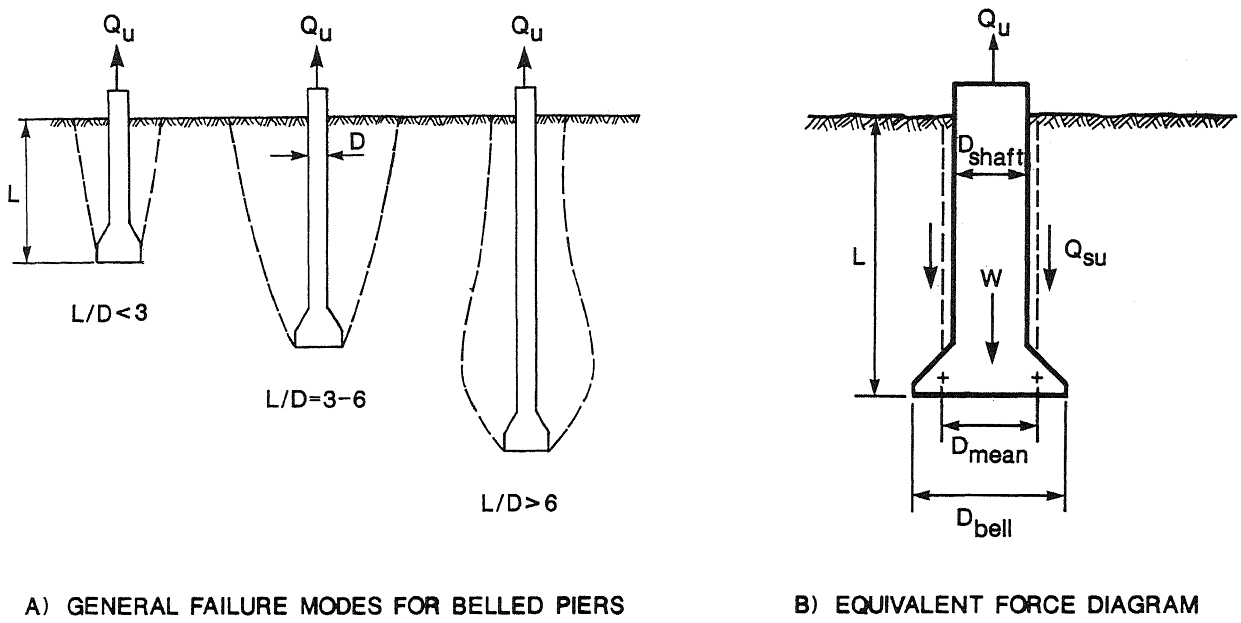


FIGURE 3. BELLED PIER IN UPLIFT

The load-deflection curves have been normalized based on the peak or ultimate capacity for drilled shafts and driven piles, as shown in Figs. 4 and 5, respectively. A method was developed to estimate the ultimate uplift capacity for test foundations where the peak uplift resistance was not reached during field load tests by using these normalized curves. The measured uplift load at small deflections was compared to the normalized uplift curves based on the type of foundation and embedded length to diameter (L/D) ratio. The ultimate uplift capacity could then be estimated using procedures given by the author (1987) for use in this evaluation.

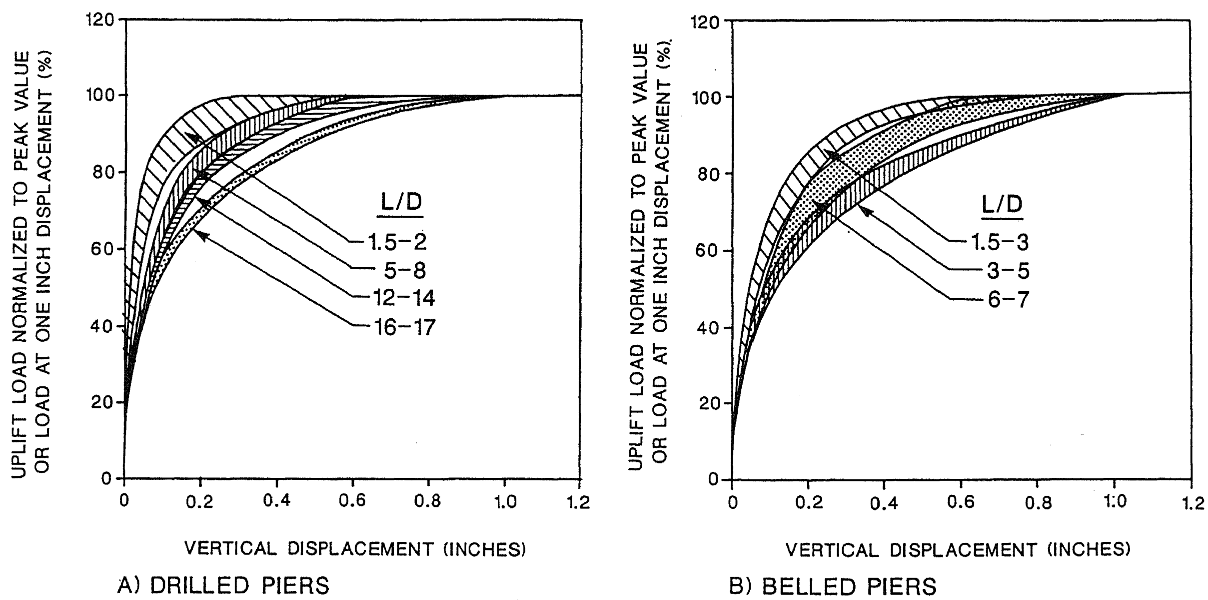
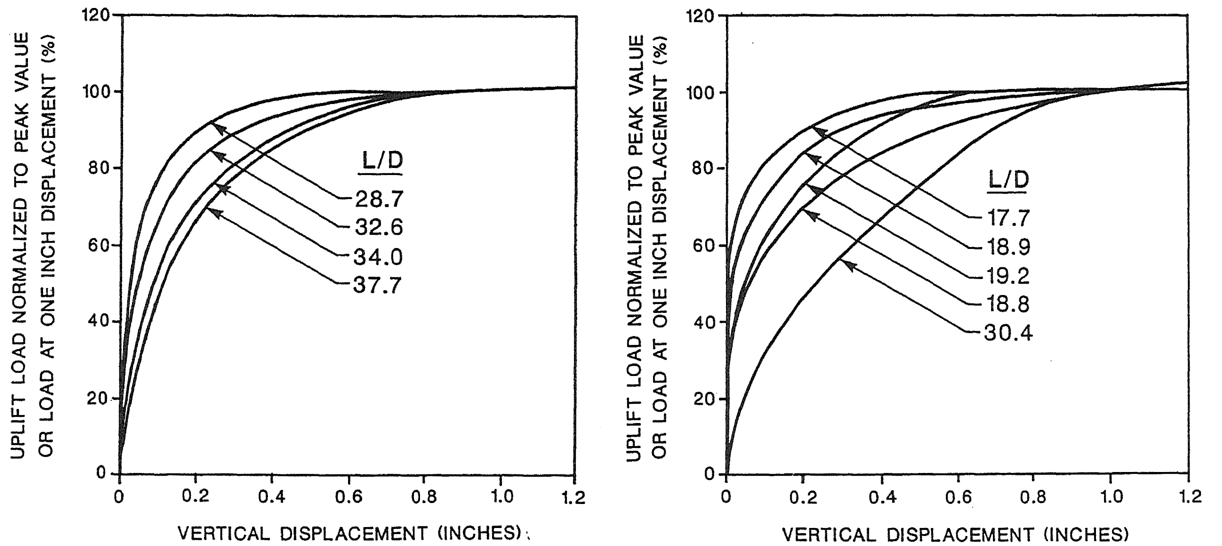


FIGURE 4. NORMALIZED UPLIFT LOAD RELATIONSHIP FOR CAST-IN-PLACE CONCRETE DRILLED SHAFTS (1 INCH=2.54 CM)



A) CONCRETE SQUARE PILES

B) RAYMOND STEEL STEP-TAPERED PILES

FIGURE 5. NORMALIZED UPLIFT LOAD RELATIONSHIP FOR DRIVEN PILES (1 INCH=2.54 CM)

Belled piers typically require larger displacements to mobilize the ultimate uplift capacity, due to the "breakout cone" failure surface, for piers with L/D ratios less than six. Also, uplift tests on slender steel and concrete driven piles in sand and clay soils indicate that the peak uplift capacity occurred at vertical deflections greater than one inch (2.5 cm). Since most structures are not designed to accommodate these large foundation displacements, the use of a deflection criteria to evaluate the design foundation capacity was incorporated.

FIELD UPLIFT LOAD TEST RESULTS

Field uplift load tests were performed on 16 drilled piers and 13 belled piers using cast-in-place concrete construction, as well as 10 prestressed concrete driven piles. The field load test results are given in Tables 1, 2 and 3 for the drilled piers, belled piers and driven piles in granular soils, respectively. The field load test results on drilled shafts in cohesive soils are listed in Table 4.

TABLE 1 RESULTS OF UPLIFT TESTS ON DRILLED PIERS IN GRANULAR SOILS

Location	Pile Length (Feet)	Pile Dia. (Feet)	L/D	Uplift Capacity Total (Kips)	Uplift Capacity Net (Kips)	Vert. Defl. (Inch)	CPT Capacity[1] $f_{s<f_l}$ (Kips)	CPT Capacity[1] Actual (Kips)	K_{soil}[1] $f_{s<f_l}$	K_{soil}[1] Actual	Ground Water (Feet)	CPT Capacity[2] $f_{s<f_l}$ (Kips)	CPT Capacity[2] Actual (Kips)	K_{sand}[2] $f_{s<f_l}$	K_{sand}[2] Actual
SCE Mira Loma-Serrano 500 kV T/L															
Site 17	25.0*	1.50*	16.7*	162.0	155.4	1.00	95.5	111.8	1.63	1.39	NE	134.7	152.0	1.15	1.02
SCE Devers-Palo Verde 500 kV T/L															
Site 4018	8.0	4.21	1.9	106.0	89.4	0.56	16.8	40.0	--	--	NE	134.8	272.1	--	--
(Slurry)				(110.5)	(93.8)	(1.00)			5.58	2.35				0.70	0.34
Site 4722	22.8	1.75	13.0	103.0	96.7	0.33	109.9	110.0	0.88	0.88	11	145.7	147.2	0.66	0.66
				100.0		0.45									
Site 4725	34.0	2.85	11.9	124.0	101.0	0.05	306.0	349.3	--	--	10	389.7	439.3	--	--
				(258.3)	(235.3)	(1.00)			0.77	0.67				0.60	0.54
Site 4727	26.5	2.80	9.5	138.0	119.8	0.10	280.0	362.2	--	--	10	483.9	616.3	--	--
				(226.2)	(208.1)	(1.00)			0.74	0.57				0.43	0.34
Site 4739A	26.2	1.15	22.8	102.0	99.0	0.97	134.2	162.2	0.74	0.61	10	145.7	173.6	0.68	0.57
Site 4739B	22.9	1.70	13.5	120.0	114.0	0.50	144.6	166.7	0.79	0.68	10	177.5	204.4	0.64	0.56
				125.0		5.14									
Site 4739C	17.0	2.05	8.3	80.0	73.0	0.53	106.2	128.2	0.69	0.57	10	162.0	194.7	0.45	0.37
				45.0		7.40									
Site 4747	22.0	1.75	12.6	130.0	123.9	0.65	133.5	138.4	0.93	0.90	15	180.4	187.0	0.69	0.66
				73.0		10.00									
LADWP Intermountain Power Project T/L															
Baker Site															
Pile No. 3	14.0	2.25	6.2	190.0	181.7	0.40	98.1	162.0	--	--	NE	223.1	355.9	--	--
				(200.0)	(191.7)	(1.00)			1.95	1.18				0.86	0.54
Pile No. 4	9.0	2.21	4.1	98.0	92.8	0.58	41.6	72.4	--	--	NE	139.8	224.0	--	--
				(100.0)	(94.8)	(1.00)			2.28	1.31				0.68	0.42

Notes: Numbers in parenthesis are based on estimated ultimate uplift capacity at a vertical deflection of one inch.
*Pier dimensions based on design sheets and were not verified in field. NE = Not encountered.
1) Values computed using equations 4 and 5, 2) Values computed using equations 6 and 7.
(1.0 kip = 4.45 kN, 1 Foot = 30.48 cm, 1 Inch = 2.54 cm)

TABLE 2. RESULTS OF UPLIFT TESTS ON BELLED PIERS IN GRANULAR SOILS

Location	Pile Length (Feet)	Pile Dia. (Feet)	Bell Dia. (Feet)	L/D	Uplift Capacity Total (Kips)	Uplift Capacity Net (Kips)	Vert. Defl. (Inch)	CPT Capacity[1] $f_s<f_1$ (Kips)	CPT Capacity[1] Actual (Kips)	K_{soil}[1] $f_s<f_1$	K_{soil}[1] Actual	Ground Water (Feet)	CPT Capacity[2] $f_s<f_1$ (Kips)	CPT Capacity[2] Actual (Kips)	K_{sand}[2] $f_s<f_1$	K_{sand}[2] Actual
SCE Magunden-Pastoria 220 kV T/L																
Site M10-T4	7.7	2.00	3.7	3.8	65.0	61.4	1.00	16.7	17.1	3.68	3.59	NE	88.3	91.8	0.70	0.67
Site M13-T1	14.0	2.00	3.8	7.0	56.0	49.4	0.02	51.6	58.5	--	--	NE	149.5	171.8	--	--
					(168.0)	(161.4)	(1.00)			3.13	2.76				1.08	0.94
Site M14-T4	14.0	2.00	3.0	7.0	74.0	67.4	0.13	22.0	22.0	--	--	NE	47.8	47.8	--	--
					(129.8)	(123.2)	(1.00)			5.60	5.60				2.58	2.58
Site M20-T3	14.0	2.00	3.7	7.0	74.0	67.4	1.00	44.2	46.0	1.52	1.47	NE	124.7	129.6	0.54	0.52
SCE Mira Loma-Serrano 500 kV T/L																
Site 4XX	10.5*	1.50*	3.0*	7.0*	102.0	99.2	0.75	40.8	43.8	2.44	2.27	NE	94.2	100.0	1.06	1.00
Site 14AX	10.5	1.50	3.0	7.0	79.0	76.2	1.00	27.5	28.0	2.77	2.72	NE	73.2	74.8	1.04	1.02
Site 29X	17.0*	2.00*	3.5	8.5*	196.0	188.0	0.22	86.2	125.8	--	--	NE	204.4	306.3	--	--
					(297.0)	(289.0)	(1.00)			3.35	2.30				1.41	0.94
Site 159	9.5	2.00	3.5*	4.8	206.0	201.5	0.14	40.0	70.0	--	--	NE	145.0	325.0	--	--
					(381.5)	(377.0)	(1.00)			9.43	5.39				2.60	1.16
SCE Devers-Palo Verde 500 kV T/L																
Site 4107A	8.0	3.13	5.0	2.6	93.6	84.4	0.60	31.4	136.6	2.69	0.62	NE	205.2	677.2	0.41	0.12
Site 4107B	7.5	2.76	5.0	2.7	124.0	117.3	0.56	27.7	112.5	4.23	1.04	NE	176.8	543.0	0.66	0.22
LADWP Intermountain Power Project T/L																
Caliente Site																
Pile No. 4	7.2	2.17	3.0	3.3	145.0	141.0	0.38	27.8	105.1	--	--	NE	132.6	501.8	1.34	0.35
					(181.3)	(177.3)	(1.00)			6.38	1.69					
Pile No. 1	9.4	2.17	3.0	4.3	200.0	194.8	0.26	45.4	154.0	--	--	NE	171.7	607.9	1.68	0.48
					(294.1)	(288.9)	(1.00)			6.36	1.88					

Notes: Numbers in parenthesis are based on estimated ultimate uplift capacity at a vertical deflection of one inch.
*Pier dimensions based on design sheets and were not verified in field. NE = Not Encountered.
1) Values computed using equations 4 and 5, 2) Values computed using equations 6 and 7.
(1.0 kip = 4.45 kN, 1 Foot = 30.48 cm, 1 Inch = 2.54 cm)

TABLE 3. RESULTS OF UPLIFT TESTS ON DRIVEN CONCRETE PILES IN GRANULAR SOILS

Location	Pile Length (Feet)	Pile Dia. (Feet)	L/D	Uplift Capacity Total (Kips)	Uplift Capacity Net (Kips)	Vert. Defl. (Inch)	CPT Capacity[1] $f_s<f_1$ (Kips)	CPT Capacity[1] Actual (Kips)	K_{soil}[1] $f_s<f_1$	K_{soil}[1] Actual	Ground Water (Feet)	CPT Capacity[2] $f_s<f_1$ (Kips)	CPT Capacity[2] Actual (Kips)	K_{sand}[2] $f_s<f_1$	K_{sand}[2] Actual
SCE Metrology Laboratory															
Pile No. 1	28.7	1.00	28.7	70.0	66.4	0.94	88.0	90.6	0.75	0.73	14	114.2	120.4	0.58	0.55
Pile No. 2	34.0	1.00	34.0	101.0	97.0	0.99	132.4	135.6	0.73	0.72	14	158.6	165.4	0.61	0.59
Pile No. 3	37.7	1.00	37.7	138.0	133.7	1.03	148.4	171.5	0.90	0.78	14	174.5	203.2	0.77	0.66
SCE Large Apparatus Repair Shop Facility															
Pile No. 2	50.0	1.17	42.9	217.0	210.2	0.65	292.4	344.0		--	10	309.5	363.7	--	--
				(230.9)	(224.0)	(1.00)			0.77	0.65				0.72	0.62
Pile No. 3	49.5	1.17	42.4	215.0	208.1	0.66	272.5	351.4	--	--	10	283.1	362.1	--	--
				(228.7)	(221.8)	(1.00)			0.81	0.63				0.78	0.61
Pile No. 5	58.0	1.17	49.7	212.0	203.8	0.65	356.6	457.3	--	--	10	374.4	476.8	--	--
				(235.5)	(227.3)	(1.00)			0.64	0.50				0.61	0.48
Pile No. 7	57.0	1.17	48.9	210.0	201.7	0.59	371.3	515.7	--	--	10	394.2	541.3	--	--
				(238.6)	(230.3)	(1.00)			0.62	0.45				0.58	0.43
SCE Devers-Palo Verde 500 kV T/L															
Site 4731A	38.0	1.17	32.6	120.0	113.8	0.80	325.9	380.2	0.35	0.30	15	364.2	431.9	0.31	0.26
Site 4731B	40.0	1.17	34.3	150.0	143.9	0.10	348.4	406.2	--	--	15	386.7	457.9	--	--
				(254.2)	(248.1)	(1.00)			0.71	0.61				0.64	0.54
Site 4735	40.0	1.17	34.3	90.0	84.2	0.10	345.8	417.7	--	--	15	378.8	453.7	--	--
				(152.5)	(146.4)	(1.00)			0.42	0.35				0.39	0.32

Notes: Numbers in parenthesis are based on estimated ultimate uplift capacity at a vertical deflection of one inch.
1) Values computed using equations 4 and 5, 2) Values computed using equations 6 and 7.
(1.0 kip = 4.45 kN, 1 Foot = 30.48 cm, 1 Inch = 2.54 cm)

The total uplift capacities in these tables represent the peak resistance from the load tests at displacements less than one inch or the estimated ultimate capacity at a vertical deflection of one inch (2.5 cm) based on the normalized curves in Figs. 4 and 5. The net uplift capacity corresponds to the side resistance along the soil-shaft interface.

FRICTIONAL CAPACITY PREDICTION USING CPT DATA

The CPT soundings provide tip resistance and side friction values of subsurface materials with associated friction ratios. An electric cone penetrometer was used at all sites in this study. The computation of shaft friction using CPT data was described by Schmertmann (1978) using the relationship:

$$Q_{sp} = K_{s,c}\left[\sum_{\ell=0}^{8D} (\ell/8D)\,(f_s)(A_s) + \sum_{8D}^{L} (f_s)(A_s)\right] \qquad (4)$$

where Q_{sp} = Predicted side friction resistance using CPT data, $K_{s,c}$ = f_s correlation factors - K_s in sand layers, K_c in clay layers, ℓ = Depth to f_s value considered, D = Shaft diameter, f_s = Unit local

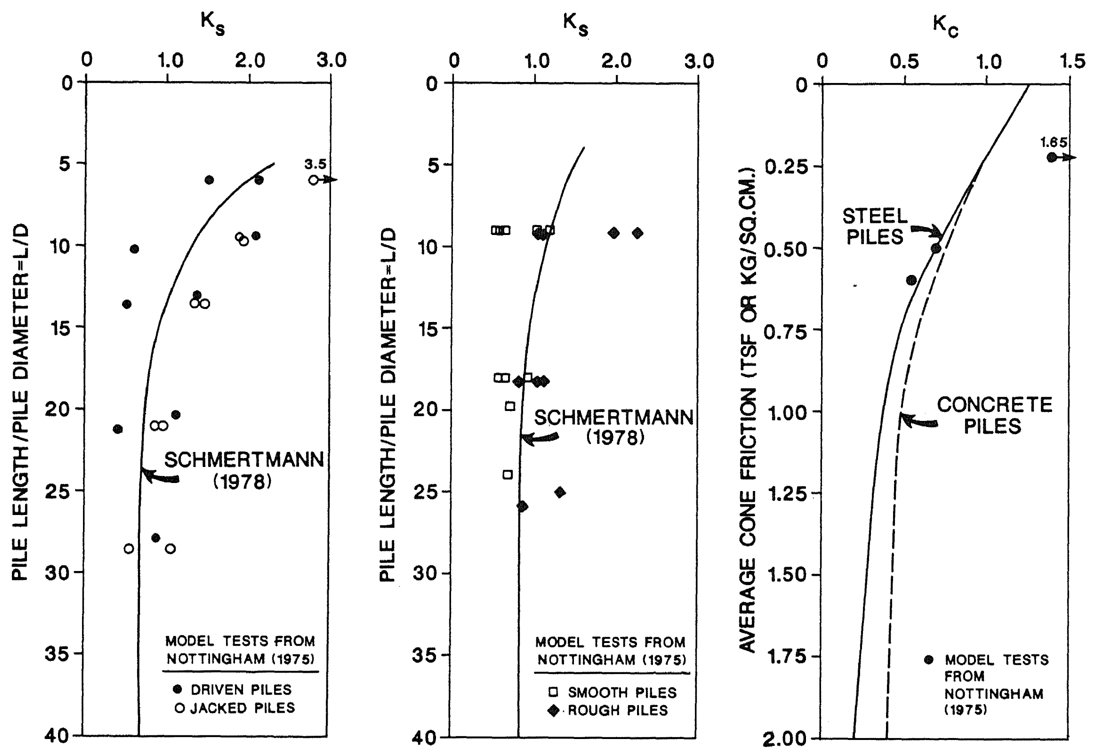
TABLE 4. RESULTS OF UPLIFT TESTS ON DRILLED SHAFTS IN COHESIVE SOILS

Location	Pier Length (Feet)	Pier Dia. (Feet)	Bell Dia. (Feet)	L/D	Uplift Capacity Total (Kips)	Uplift Capacity Net (Kips)	Vert. Defl. (Inch)	Ground Water (feet)	Ave. Shaft Friction (Ksf)	Ave. Cone Friction (Ksf)	Adherence Coeff.-M [1]
SCE Mira Loma-Serrano 500 kV T/L											
Site 27	9.5	1.5	3.0	6.3	119.0	116.5	0.65	NE	1.95	2.36	0.83
					(126.5)	(124.0)	(1.00)		2.08		0.88
LADWP Intermountain Power Project T/L											
Delta Site											
Pile No. 1	9.4	2.17	--	4.3	107.0	101.7	0.50	18	1.59	4.02	0.39
					(110.0)	(104.7)	(1.00)		1.64		0.41
Pile No. 2	9.4	2.08	--	4.5	105.0	100.2	0.62	18	1.63	4.02	0.41
					(108.0)	(103.2)	(1.00)		1.68		0.42
Pile No. 3	14.4	2.17	--	6.6	160.0	152.0	0.60	18	1.55	4.30	0.36
					(168.0)	(160.0)	(1.00)		1.63		0.38
Alamo Site											
Pile No. 4	8.9	2.13	--	4.2	200.0	195.2	0.32	NE	--	18.50	--
					(213.0)	(208.2)	(1.00)		3.50		0.19
Pile No. 3	13.9	2.13	--	6.5	200.0	192.6	0.16	NE	--	23.58	--
					(256.4)	(249.0)	(1.00)		2.68		0.11

Notes: Numbers in parenthesis are based on estimated ultimate uplift capacity at a vertical deflection of one inch.
NE = not recommended. 1) Values computed using equation 8. (1.0 kip = 4.45 kN, 1 Foot = 30.48 cm, 1 Inch = 2.54 cm)

side friction resistance from CPT data, A_s = Pile-soil contact area per f_s depth interval, and L = Total embedded length of pile.

The K_s and K_c values are shown in Fig. 6 from Schmertmann (1978). K_s represents a correction factor to be utilized with granular soils and is dependent on embedded length, pile diameter and type of material. The correction factor, K_s, for granular materials was derived from load tests on smooth and rough model piles by Nottingham (1975). The rough piles produced much higher K_s values than results from smooth pile tests. The stress history and coefficient of earth pressure at rest, K_0, may be



A) STEEL PIPE PILES IN SAND
B) CONCRETE SQUARE PILES IN SAND
C) STEEL AND CONCRETE PILES IN CLAY

FIGURE 6. SIDE FRICTION CORRELATION FACTORS FOR ELECTRICAL PENETROMETER

evaluated using Standard Penetration Test (SPT) blowcounts, CPT results or pressuremeter tests, as described by Kulhawy, et al. (1984).
The K_c value represents a correction factor for cohesive soils which is primarily dependent upon the shear strength of the fine grained soil. Many axial load tests in clay soils have been performed to evaluate the adhesion factors for soft to stiff clays on wood, steel and concrete piles. The correction factor, K_c, for clay soils shown in Fig. 6 was based on test results from Tomlinson (1957) and others where the soil adhesion is compared to the undrained shear strength of the clay soils, as well as limited model tests by Nottingham 1975).

The predicted frictional resistance, Q_{sp}, from CPT data was computed using Eq. 4 with K_c values from Fig. 6 and K_s values set equal to one for each clay and sand layer, respectively. The pile-soil contact area for each interval was calculated using the average diameter for drilled piers and driven piles and mean diameter for belled piers. The side friction values, f_s, were selected for two cases: (1) $f_s \le f_l$ where f_l = 1.2 tsf (kg/sq.cm.) and (2) f_s = Actual values from CPT data. The limiting side friction value, f_l, was proposed by Schmertmann based on methods used by Dutch engineers.

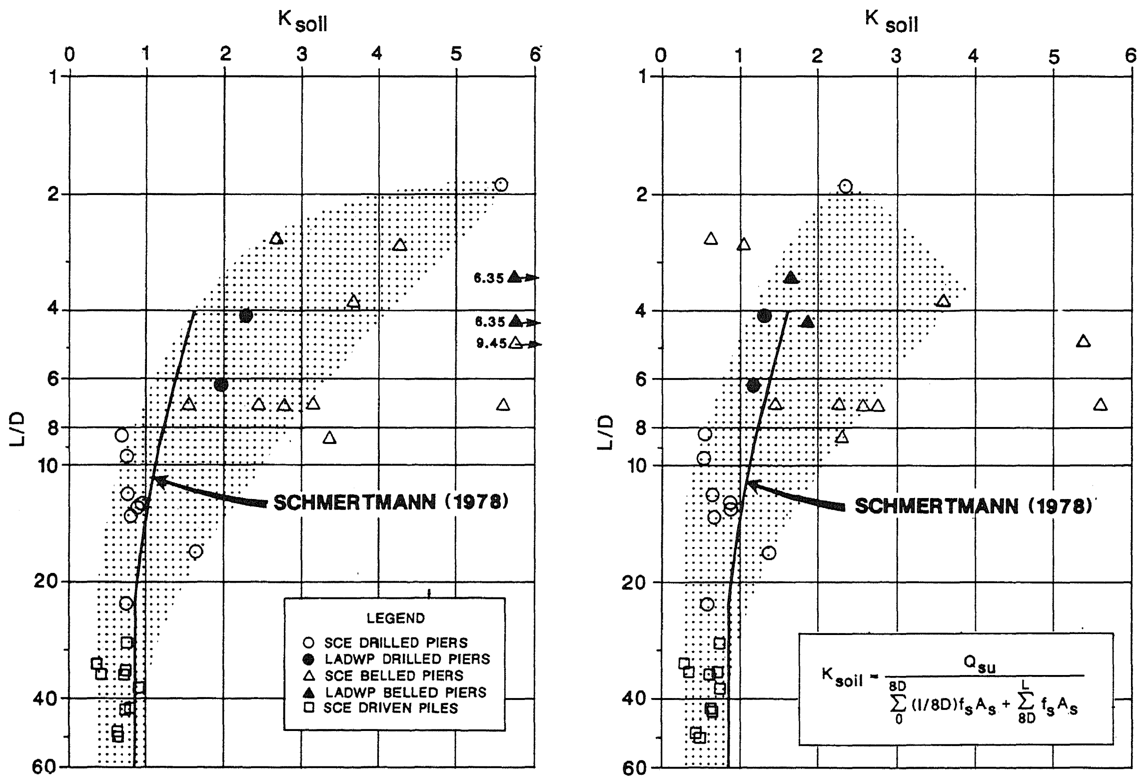
The peak and estimated ultimate uplift capacities from field load tests were compared to the predicted side resistance capacity from Eq. 4 using the following relationship:

$$K_{soil} = Q_{su} / \left[\sum_{\ell=0}^{8D} (\ell/8D)\,(f_s)(A_s) + \sum_{8D}^{L} (f_s)(A_s) \right] \qquad (5)$$

where K_{soil} = Correlation factor for granular materials and Q_{su} = Ultimate side friction resistance from uplift load tests. The K_{soil} values are listed in Tables 1, 2 and 3 and are shown graphically in Fig. 7 along with the original design curves from Fig. 6. For the case with $f_s \le f_l$ = 1.2 tsf, the field uplift test results correlate well with Schmertmann's curve for drilled piers and driven piles with L/D ratios greater than 10. Drilled shafts with L/D ratios less than 10 yield much higher actual capacities than predicted based on model tests by Nottingham (1975). When the actual cone friction values are incorporated into Eq. 4, the field test results are lower than the original design curve for foundations with L/D ratios greater than 10 and somewhat higher for drilled shafts with L/D ratios less than nine. These conclusions from SCE field load tests are consistent with results from Horvitz, et al. (1981) and others.

REVISED DESIGN METHODOLOGY

The predicted side frictional resistance of drilled shafts and driven piles from Eq. 4 incorporates an $\ell/8D$ reduction factor to the side friction values from CPT soundings. Uplift load tests on model piles



A) WITH $f_s \le f_l$ =1.2 TSF OR KG/SQ.CM.

B) WITH f_s=ACTUAL VALUES

FIGURE 7. CPT SIDE FRICTION CORRECTION FACTOR, K_{soil}, FOR SANDS FROM FIELD UPLIFT LOAD TESTS

in granular soils by Das and Seeley (1975), Das and Rozendal (1983), and Meyerhof (1973) indicated that the unit skin friction along the pile increases linearly to a limiting value at a critical depth. This critical depth is dependent upon the soil relative density, shear strength, pile material and roughness along the soil-pile interface. Field axial load tests on circular steel pipe piles conducted by Vesic (1970) and others show that distribution of skin friction along pile shafts is generally parabolic. The limiting skin friction values occurred at embedded length to diameter (L/D) ratios from 6 to 14 in model tests and 2 to 30 in field tests.

Field uplift loads tests performed by SCE over the past 50 years indicate that the average skin friction decreases as the L/D ratio becomes larger in granular soils. These test results were presented by the author (1987) and are shown graphically in Fig. 8. This trend also occurs for drilled shafts in cemented sand and gravels as well as rock materials.

Granular soils - Since field uplift load tests on full scale foundations in granular soils show that a reduction in skin friction for drilled piers with L/D ratios less than 8 is not warranted, the computed side friction resistance in granular soils using CPT data may be obtained from the relationship:

$$Q_{ss} = K_{sand} \sum_{0}^{L} (f_s)(A_s) \qquad (6)$$

where Q_{ss} = Total side friction resistance in sands and K_{sand} = Correlation factor for granular soils. The K_{sand} values from field load tests were computed from the relationship:

$$K_{sand} = Q_{su} / \sum_{0}^{L} (f_s)(A_s) \qquad (7)$$

The Q_{ss} and K_{sand} values from equations 6 and 7, respectively, are given in Tables 1, 2 and 3. Again, two cases were evaluated regarding the CPT side friction values using a limiting condition and actual records as previously discussed. The K_{sand} values are shown graphically versus the L/D ratio of each foundation in Fig. 9. The range of K_{sand} values from Eq. 7 is much less than corresponding K_{soil} values from Eq. 5, indicating that the use of Eq. 6 to predict the side friction resistance with CPT data gives more consistent results as compared to the field uplift test results.

Cohesive Soils - In cohesive soils, field uplift load test results on drilled shafts by SCE and LADWP are tabulated in Table 4. The average skin friction value was computed for each pier from the field load test results using the shaft diameter for drilled piers and the mean diameter for belled piers. The average side friction value from CPT data was calculated at each test site with the adherence coefficient obtained from the relationship:

$$m = f_{ave}/\bar{f}_s \qquad (8)$$

where m = Adherence coefficient in cohesive soils, f_{ave} = Average skin friction along shaft from field load tests, and $\bar{f}_s$ = Average side friction value from CPT data. These values are listed in Table 4 and are

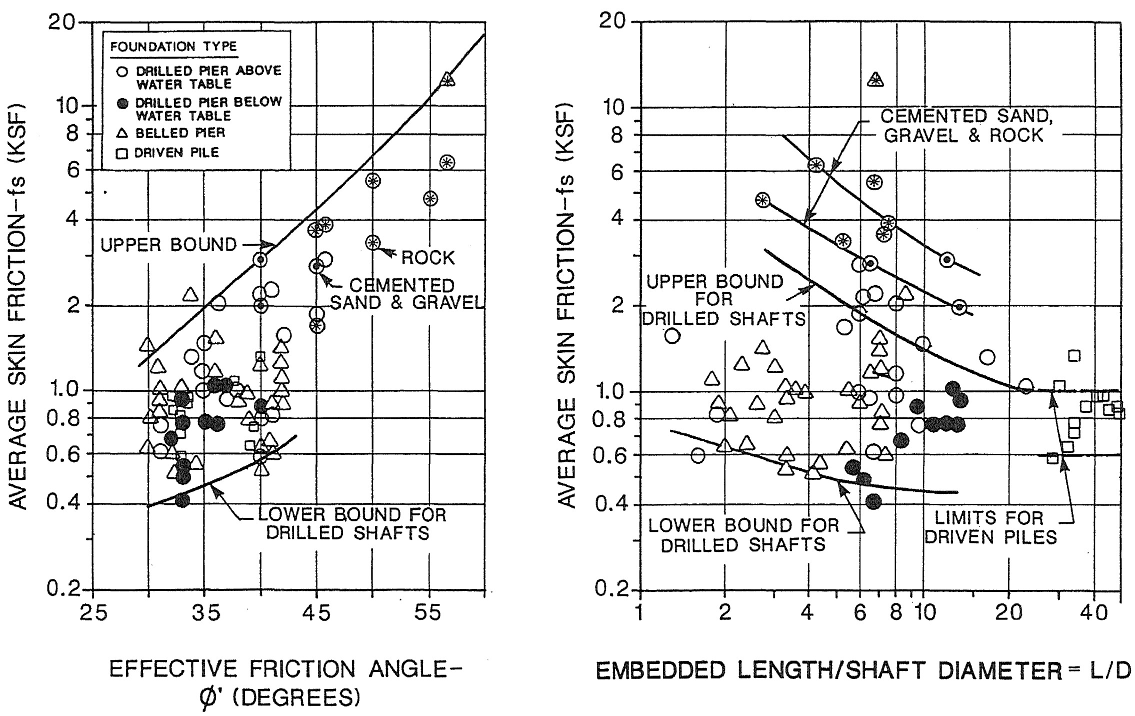
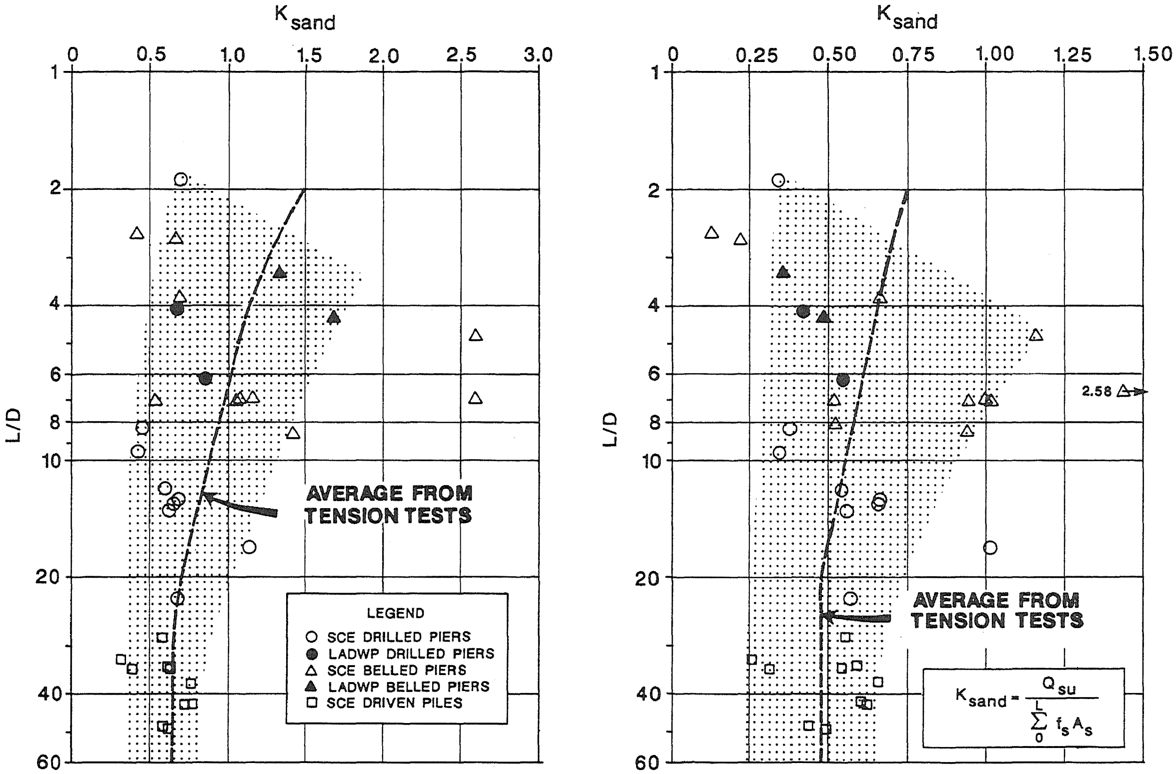


FIGURE 8. VARIATION OF AVERAGE SKIN FRICTION, fs, WITH SHEAR STRENGTH AND FOUNDATION GEOMETRY FROM SCE UPLIFT LOAD TESTS (1 KSF=0.5 KG/SQ.CM.)



A) WITH $f_s \leq f_l = 1.2$ TSF OR KG/SQ.CM.

B) WITH f_s = ACTUAL VALUES

FIGURE 9. CPT SIDE FRICTION CORRECTION FACTOR, K_{sand}, FOR SANDS FROM FIELD UPLIFT LOAD TESTS

shown graphically in Fig. 10. Field load test results from Tumay, et al., (1983) and O'Neill (1986) in clay soils are also shown in Fig. 10 and compare favorably with the SCE and LADWP test results. Thus, for drilled shafts and driven piles in cohesive soils, the side friction resistance, Q_{sc}, can be computed from the relationship:

$$Q_{sc} = \sum_0^L (m)(f_s)(A_s) \qquad (9)$$

General Equation - At sites where the soil conditions are predominately sand with some intermixed clay layers, the general equation to obtain the total side friction resistance, Q_{st}, is as follows:

$$Q_{st} = K_{sand} \sum_0^L (m)(f_s)(A_s) \qquad (10)$$

where: 1) In granular soils, m = 1.0 and K_{sand} is selected from Fig. 9 based on the embedded length to shaft diameter (L/D) ratio, and 2) In cohesive soils, K_{sand} = 1.0 and m is selected from Fig. 10 based on the average side friction value from CPT data for each distinct clay layer.

CONCLUSIONS

A performance evaluation was made based on field uplift load test performed by SCE over the past 50 years. Normalized curves were developed for drilled shafts and driven piles, shown in Figs. 4 and 5, respectively, based on these uplift test results. The ultimate uplift capacity was selected based on an allowable vertical displacement of one inch (2.5 cm) from field load tests where the maximum uplift resistance was obtained. These curves may be used to predict the uplift capacity of drilled shafts and driven piles for various embedded length to diameter (L/D) ratios of the foundation.

Field uplift load tests were performed on 16 drilled piers, 13 belled piers and 10 concrete driven piles by SCE and LADWP at sites with CPT soundings. These test results are given in Tables 1, 2 and 3 for granular soils and Table 4 for cohesive materials. The measured uplift capacity or estimated ultimate uplift capacity from normalized curves were used to obtain the side friction resistance of each test foundation.

A revised design methodology was presented to compute the total side friction resistance of foundations using CPT data. The general relationship is given by Eq. 10 for granular and cohesive soils with associated side friction correlation factors. The correlation factor for granular soils, K_{sand}, was obtained from Eq. 7 and is shown in Fig. 9 versus the L/D ratio of test foundations. In cohesive soils, the adherence

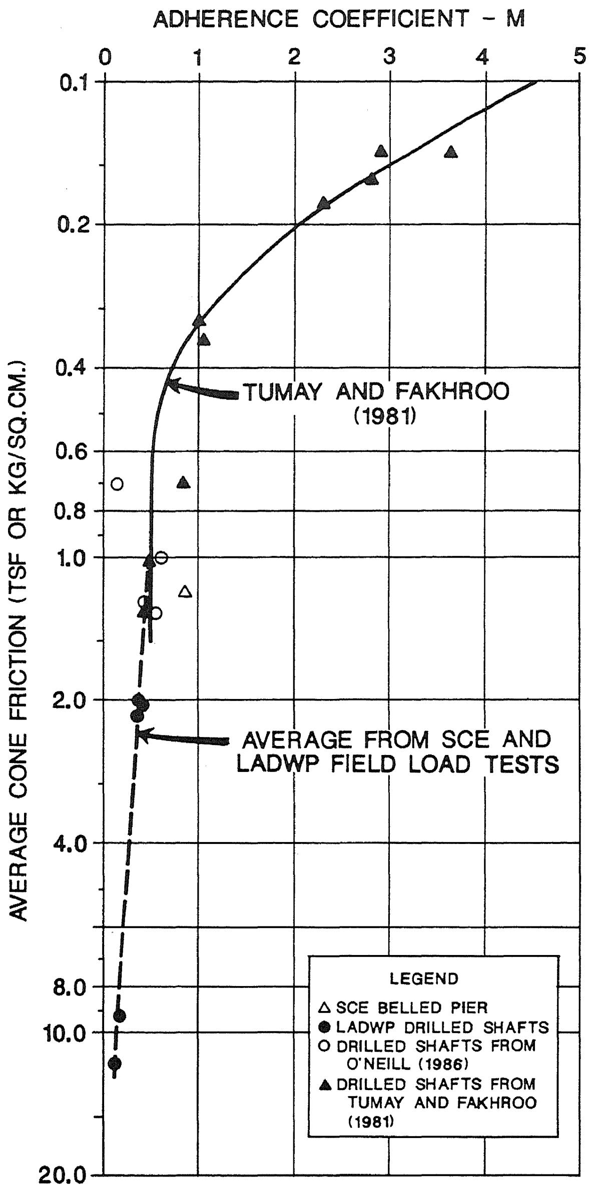


FIGURE 10. CPT SIDE FRICTION CORRECTION FACTOR, M, FOR COHESIVE SOILS

coefficient- m shown in Fig. 10, should be combined with the side friction value from CPT soundings to compute the actual adhesion along the embedded area.

The actual side friction values recorded from electric CPT soundings yield better correlations with the field uplift test results than by using a limiting value of 1.2 tsf (kg./sq.cm.), as recommended by Schmertmann (1978). The K_{sand} values in Fig. 9 range from 0.12 to 1.16 using actual f_s data while a much greater variance, from 0.31 to 2.60, was obtained for K_{sand} values with a limiting value for f_s. Also, no reduction factors should be applied to the computed side friction resistance for drilled shafts with L/D ratios of 8 or less, as shown in Figure 8, based on SCE field load test results.

REFERENCES

Briaud, J. C., Pacal, A. J. and Shively, A. W., "Power Line Foundation Design Using the Pressuremeter," International Conference on Case Histories in Geotechnical Engineering, St. Louis, 1984.

Das, B. M. and Seeley, G. R., "Uplift Capacity of Buried Model Piles in Sand," Journal of the Geotechnical Engineering Division, ASCE, No. GT10, 1975, pp. 1091-1094.

Das, B. M. and Rozendal, D. B., "Ultimate Uplift Capacity of Piles in Sand," Transportation Research Record No. 945, 1983, pp. 40-45.

Horvitz, G. E., Stettler, D. R. and Crowser, J. C., "Comparison of Predicted and Observed Pile Capacity", Cone Penetration Testing and Experience, ASCE STP, October 1981, pp. 413-433.

Kulhawy, F. H. Trautmann, C. H., Beech, J. F., O'Rourke, T. D., McGuide, W., Wood, W. A. and Capono, C., "Transmission Line Structure Foundations for Uplift-Compression Loading," Report EL-2870, Electric Power Research Institute, Palo Alto, CA, November, 1984

Kulhawy, F. H. "Drained Uplift Capacity of Drilled Shafts," Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, California, August, 1985.

Meyerhof, G. G., "The Uplift Capacity of Foundations Under Oblique Loads," Canadian Geotechnical Journal, Vol. 10, No. 1, 1973, pp. 64-70.

Nottingham, L. C., "Use of Quasi-Static Friction Cone Penetrometer Data to Predict Load Capacity of Displacement Piles," Ph.D. Dissertation to the Department of Civil Engineering, University of Florida, 1975.

Reese, L. C. Touma, F. T., and O'Neill, M. W., "Behavior of Drilled Piers Under Axial Loading", Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, No. GT5, May 1976.

Schmertmann, J. H., "Guidelines for Cone Penetration Test, Performance and Design," U.S. Department of Transportation, Federal Administration, Offices of Research and Development, Washington, D.C., Publication No. FHWA-TS-T8-209, July 1978.

Tomlinson, M. J., "The Adhesion of Piles Driven in Clay Soils," Proceeding of the 4th International Conference on Soil Mechanics and Foundation Engineering, London, 1957, Vol. 2, p. 66.

Tucker, K. D., "Uplift Capacity of Drilled Shafts and Driven Piles in Granular Materials", Foundation for Transmission Line Towers, ASCE STP, April 1987, pp. 142-159.

Tumay, M. T. and Fakhroo, M., "Pile Capacity in Soft Clays Using Electric QCPT Data,"Cone Penetration Testing and Experience, ASCE STP, October, 1981, pp. 434-455.

Vesic, A. S., "Tests on Instrumented Piles, Ogeechee River Site," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 96, No. SM2, Proc. Paper 7170, March, 1970, pp. 561-584.